NWS FLDWAV Model: The Replacement of DAMBRK for Dam-Break Flood Prediction

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INTRODUCTION

The NWS DAMBRK model, which has been used by the National Weather Service (NWS) and many in the general engineering community since the late 1970's for predicting the flooding from an actual or hypothetical dam failure, is scheduled for replacement by the FLDWAV model. The FLDWAV model has been undergoing development and testing intermittently for several years (Fread and Lewis, 1988) within the NWS Hydrologic Research Laboratory. It is planned to be released during the fall of 1993 to a few selected users within NWS and outside the agency for additional testing of the model. A general release of the FLDWAV model is scheduled for the summer of 1994.

FLDWAV, like DAMBRK, computes the outflow hydrograph from a dam due to spillway, overtopping, and/or dam-breach outflows. The resulting floodwave is then routed through the downstream channel/valley using an implicit finite-difference numerical solution of the complete Saint-Venant equations of one-dimensional unsteady flow, along with appropriate internal boundary equations representing downstream dams, bridges, weirs, waterfalls, and other man-made/natural flow controls. The flow may be "mixed" (subcritical and/or supercritical) throughout the downstream routing reach, and the flow may vary from Newtonian (water) to non-Newtonian (mud/debris). The FLDWAV model includes the following features not found in DAMBRK: (a) the flood may occur in a system of interconnected rivers such as the main-stem river and its tributaries or through bifurcated channels and bypasses; (b) levee-overtopping/crevasse flows into and through levee-protected floodplains that may be compartmentalized by dikes and elevated roadways; (c) automatic calibration of Manning roughness coefficient for historical floods; (d) improved numerical stability features; (e) menu-driven interactive data input; and (f) color-graphic displays of model output. FLDWAV is a generalized flood routing model that can be used by hydrologists/engineers for real-time flood forecasting of dam-break floods and/or natural floods, dam-breach flood analysis for sunny-day piping or overtopping associated with the PMF flood, floodplain inundation mapping for contingency dam-break flood planning, debris flow inundation mapping, and design of waterway improvements.

FLDWAV MODEL DESCRIPTION

Governing Equations

The governing equations of the FLDWAV model are: (1) the expanded one-dimensional equations of unsteady flow originally derived by Saint-Venant; (2) an assortment of internal

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boundary equations of flow through one or more flow control structures, such as dams, located along the main-stem river and/or its tributaries; and (3) external boundary equations of known upstream/downstream discharges or water elevations which vary either with time or each other.

Expanded Saint-Venant Equations: An expanded form of the Saint-Venant equations of conservation of mass and momentum consist of the following (Fread, 1988):

$$\partial Q/\partial x + \partial s_c(A+A_o)/\partial t - q = 0 \tag{1}$$

in which Q is discharge (flow); A is wetted active cross-sectional area; A_o is wetted inactive off-channel (dead) storage area associated with topographical embayments or tributaries; s_c is a depth-dependent channel sinuosity coefficient; q is lateral flow (inflow is positive, outflow is negative); t is time; and x is distance measured along the mean flow-path of the floodplain. The conservation of momentum equation is:

$$\partial(s_m Q)/\partial t + \partial(\beta Q^2/A)/\partial x + gA(\partial h/\partial x + S_f + S_e + S_i) + L = 0 \tag{2}$$

in which s_m is another depth-dependent sinuosity coefficient, g is the gravity acceleration constant; h is the water surface elevation; L is the momentum effect of lateral flows ($L = -qv_x$ for lateral inflow where v_x is the lateral inflow velocity in the x-direction, $L = -qQ/(2A)$ for seepage lateral outflows, $L = -q\, Q/A$ for bulk lateral outflows such as flows over levees); S_f is the boundary friction slope, i.e., $S_f = |Q|Q/K^2$ in which $K = 1.49\ AD^{2/3}S^{1/2}/n$ is the total conveyance determined by summing conveyances of the left/right floodplains and the channel in which the channel conveyance is modified by the factor, $1/s_m^{1/2}$, and all conveyances are determined automatically from the data input of topwidth/Manning n versus elevation tables for cross sections of the channel and left/right floodplains; S_e is the expansion/contraction slope, i.e., $S_e = k_e/(2g) \cdot \partial(Q/A)^2/\partial x$ where k_e is the expansion/contraction loss coefficient; β is the momentum coefficient for non-uniform velocity distribution and is internally computed from the conveyances and areas of the channel and left/right floodplains and S_i is the internal viscous dissipation slope for non-Newtonian (mine tailings/mud/debris) flows, i.e.,

$$S_i = \kappa/\gamma[(b+2)Q/(AD^{b+1}) + (b+2)/(2D^b)\ (\tau_o/\kappa)^b]^{1/b} \tag{3}$$

in which $D=A/B$ where B is the wetted topwidth; κ is the apparent fluid viscosity; γ is the fluid's unit weight; τ_o is the initial shear strength of the fluid; and $b = 1/m$ where m is the exponent of a power function that represents the fluid's stress (τ_s)-rate of strain (dv/dy) relation, i.e., $\tau_s = \tau_o + \kappa(dv/dy)^m$ in which v and y are the flow velocity and depth.

Internal Boundary Equations: Locations along the main-stem and/or tributaries where the flow is rapidly varied in space and Eqs. (1-2) are not applicable, e.g. dams, bridges/road-embankments, waterfalls, short steep rapids, weirs, etc. These locations require the following internal boundary equations in lieu of Eqs. (1-2):

$$Q_i - Q_{i+1} = 0 \tag{4}$$

$$Q_i = f(h_i,\ h_{i+1},\ \text{properties of control structure}) \tag{5}$$

in which the subscripts i and i+1 indicate cross sections just upstream and downstream of the structure, respectively. For a bridge, Eq. (5) becomes:

$$Q_i = \sqrt{2g}\ C_b A_b\ (h_i - h_{i+1} + v^2/2g - \Delta h_f)^{1/2} + C_e L_e K_e (h_i - h_e)^{3/2} \tag{6}$$

in which C_b is the coefficient of flow through the bridge, A_b is the wetted cross-sectional area of the bridge opening, $v = Q_i/A_i$, Δh_f is the head loss through the bridge, C_e is the coefficient of flow over the embankment, L_e is the length of the road embankment, h_e is the elevation of the embankment crest, and K_e is a broad-crested weir submergence correction, i.e., $K_e = 1\text{-}23.8\ (r - 0.67)^3$, where $r = (h_{i+1} - h_e)/(h_i - h_e)$ if $r > 0.67$, otherwise $K_e = 1$. If the internal boundary represents a waterfalls or short steep rapids, Eq. (5) becomes:

$$Q_i = (g/B_i)^{1/2}\ A_i^{3/2} \tag{7}$$

If the flow structure is a dam, Eq. (5) becomes:

$$Q_i = K_s C_s L_s (h_i - h_s)^{3/2} + \sqrt{2g}\ C_g A_g (h_i - h_g)^{1/2} + K_d C_d L_d (h_i - h_d)^{3/2} + Q_t + Q_{br} = 0 \tag{8}$$

in which K_s, C_s, L_s, and h_s are the uncontrolled spillway's submergence correction factor, coefficient of discharge, length of spillway, and crest elevation, respectively; K_d, C_d, L_d, and h_d are similar properties of the crest of the dam; C_g, A_g, and h_g are the coefficient of discharge, area, and height of opening of a fixed or time-dependent moveable gate spillway; Q_t is a constant or time-dependent turbine discharge; and Q_{br} is a time-dependent dam breach flow, i.e.,

$$Q_{br} = C_v K_b [3.1\ b_i (h_i - h_b)^{3/2} + 2.45\ z\ (h - h_b)^{5/2}] \tag{9}$$

in which b_i is the known time-dependent bottom width of the breach, h_b is the known time-dependent bottom elevation of the breach, z is the side slope of the breach (1: vertical to z: horizontal), C_v is a velocity of approach correction factor, and K_b is a broad-crested weir submergence correction factor similar to K_e in Eq. (6). The instantaneous breach width is $b_i = b(\tau_b/\tau)^e$, in which b is the bottom width of the breach at peak outflow through the breach, τ_b is the time since beginning of the breach formation, τ is the total time for the breach to form, and e is an exponent $1 \le e \le 3$ with e usually assumed as unity (linear breach formation). Also, the instantaneous breach bottom elevation is $h_b = h_d - H_d\ (\tau_b/\tau)^e$, in which h_d is the elevation of the dam crest and H_d is the height of the dam. The final breach width (b) is related to the average breach width ($\bar{b}$), i.e., $\bar{b} = b - zH_d$. Breach properties, τ and $\bar{b}$, may be determined from empirical statistical relations or from breach simulation models, e.g. the NWS BREACH model (Fread, 1984). Statistical relations using observed breach data reported by (Froelich, 1988) are as follows:

$$\tau = 0.59\ V_r^{0.47}/(H_d)^{0.9} \tag{10}$$

$$\bar{b} = 9.5\ (V_r H_d)^{0.25} \tag{11}$$

in which V_r is the maximum reservoir volume (acre-ft). The standard error of τ is ± 0.9 hrs or ± 22 percent, and the standard error of $\bar{b}$ is ± 69 ft or $\pm$ 42 percent (Fread, 1988).

External Boundary Equations: External boundary equations which are required at the upstream and downstream extremities of the waterway may be a specified time series of discharge (a discharge hydrograph) for the upstream boundary or water elevation as in the case of a lake level or estuarial tidal fluctuation for the downstream boundary. Also, the downstream boundary can be Eq. (7), Eq. (8), an empirical rating of h and Q, or a channel control, loop-rating based on the Manning equation in which S (the dynamic energy slope) is approximated by:

$$S = -(h_N - h_{N-1})/\Delta x - (Q^{t+\Delta t} - Q)/(gA\ \Delta t) - [(Q^2/A)_N - (Q^2/A)_{N-1}]/(gA\ \Delta x) \tag{12}$$

in which Δx is the distance between the last two cross sections at the downstream boundary.

Solution Technique

In FLDWAV, the Saint-Venant Eqs. (1-2) are solved by a weighted four-point nonlinear implicit finite-difference numerical technique as described by (Fread, 1985). Substitution of appropriate simple algebraic approximations for the derivative and non-derivative terms in Eqs. (1-2) result in two nonlinear algebraic equations for each Δx reach between specified cross sections. When these are specified for each Δx reach and are combined with the external boundary equations and any necessary internal boundary equations, the combined set of equations may be solved by an iterative quadratic solution technique (Newton-Raphson) along with an efficient, compact, quad-diagonal Gaussian elimination matrix solution technique. Initial conditions required at $t=0$ are automatically obtained via a steady flow backwater solution provided within FLDWAV.

SPECIAL FEATURES OF FLDWAV

The FLDWAV model has several special features including: flow routing through a system of interconnected waterways, levee overtopping/floodplain interactions, an enhanced subcritical/supercritical mixed-flow solution algorithm, automatic calibration, and several automatic numerical stability enhancements. A brief description of each feature follows.

Flow Through System of Waterways

The FLDWAV model, unlike the DAMBRK model, can route unsteady flows occurring simultaneously in a system of interconnected waterways. Any of the waterways may contain one or more dams which control the flow and which may breach if failure conditions are reached. A river system consisting of a main-stem river and one or more principal tributaries is efficiently solved using an iterative relaxation method (Fread, 1973) in which the flow at the confluence of the main-stem and tributary is treated as the lateral inflow/outflow (q) in

Eqs. (1-2). If the river consists of bifurcations such as islands and/or complex dendritic systems with tributaries connected to tributaries, etc., a network solution technique is used (Fread, 1985), wherein three internal boundary equations conserve mass and momentum at the confluence. This system of algebraic equations uses another special compact Gaussian elimination matrix technique within FLDWAV for an efficient solution.

Levee Overtopping/Floodplain Interactions

Flows which overtop levees located along either or both sides of a main-stem river and/or its principal tributaries can be treated as lateral flow (q) in Eqs. (1-2) where the diverted lateral flow over the levee is computed as broad-crested weir flow. This overtopping flow is corrected for submergence effects if the floodplain water-surface elevation sufficiently exceeds the levee crest elevation. The overtopping flow may reverse its direction when the floodplain elevation exceeds the river water-surface elevation, thus allowing flow to return to the channel after the flood peak passes. The overtopping broad-crested weir flow is computed according to the following:

$$q = -c_\ell \, K_s \, (h - h_c)^{3/2} \tag{13}$$

where K_s is computed as in Eq. (6) except $r = (h_{fp} - h_c) / (h - h_c)$, in which c_ℓ is the weir discharge coefficient, K_s is the submergence correction factor similar to that used in Eq. (9) for dam breaches, h_c is the levee-crest elevation, h is the water-surface elevation of the river, and h_{fp} is the water-surface elevation of the floodplain. Flow in the floodplain can affect overtopping flows via the submergence correction factor, K_s. Flow may also pass from the waterway to the floodplain through a time-dependent crevasse (breach) in the levee via a breach-flow equation similar to Eq. (9). The floodplain, which is separated from the principal routing channel (river) by the levee, may be treated as: (a) a dead storage area (A_o) in the Saint-Venant equations; (b) a tributary which receives its inflow as lateral flows (the flows from the river which overtop the levee-crest) which are dynamically routed along the floodplain; and (c) the flows and water-surface elevations can be computed by using a level-pool routing method particularly if the floodplain is divided into compartments by levees (dikes) or elevated roadways located perpendicular to the river levee(s).

Supercritical/Subcritical Mixed Flow

Flow can change with either time or distance along the routing reach from supercritical to subcritical while passing through critical flow, or conversely. This "mixed flow" requires special treatment to prevent numerical instabilities in the solution of the Saint-Venant equations. FLDWAV addresses this difficulty by using a concept based on avoiding the use of the Saint-Venant equations at the point where mixed flow occurs. An enhanced mixed flow algorithm automatically subdivides the total routing reach into sub-reaches wherein only subcritical or supercritical flows occur. The transition locations where flow changes from subcritical to supercritical or vice versa are treated as boundary conditions thus avoiding the application of the Saint-Venant equations to the transition flow and subsequent numerical solution difficulties. The mixed-flow algorithm has two components, one for obtaining the initial condition of discharge and water elevation at t=0 and another which functions during the unsteady flow solution. The Froude number (Fr) is used to determine the supercritical reaches, for which $Fr > 1$. At each time step, the solution commences with the most upstream sub-reach, and proceeds sub-reach by sub-reach in the downstream direction. Hydraulic jumps are allowed to move upstream (downstream) at the end of a time step according to the relative values of supercritical (subcritical) sequent depth and the adjacent downstream subcritical (upstream supercritical) depth. Smaller computational distance steps (Δx) are required in the vicinity of the transition locations where either critical flow or a hydraulic jump occurs.

Automatic Calibration

An option within FLDWAV allows the automatic determination of the Manning n such that the difference between computed water surface elevations (stage hydrographs) and observed hydrographs is minimized. The Manning n can vary with either flow or water elevation and with each sub-reach separated by water level recorders. The algorithm (Fread and Smith, 1978) for efficiently accomplishing this is applicable to a single multiple-reach river or a main-stem river and its principal tributaries. FLDWAV also provides an option (Fread and Lewis, 1986) for determining optimal n values which may for some applications eliminate the need for time-consuming preparation of detailed cross-sectional data. Approximate cross sections represented by a 2-parameter power function for the channel and another for the floodplain can be estimated from topographical maps or a few site visits. The estimated cross sections are automatically adjusted as necessary to enable optimized n values to fall within user-specified min-max ranges. Also, specific cross-sectional properties at key sections (bridges, natural constrictions, etc.) can be utilized wherever necessary.

Numerical Stability Enhancements

Automatic Interpolation: The FLDWAV model can automatically provide linearly interpolated cross-sections at a user specified spatial resolution in order to increase the spatial frequency at which solutions to the Saint-Venant equations are obtained. This is often required for purposes of attaining numerical stability when (a) routing very sharp-peaked hydrographs such as those generated by breached dams, (b) when adjacent cross sections either expand or contract by more than about 50 percent, and (c) where mixed flow changes from subcritical to supercritical or vice versa.

Automatic Selection of $\Delta x/\Delta t$ Computational Parameters: The FLDWAV model can use either a specified Δt computational time step or one that is automatically determined to best suit the most rapidly rising hydrograph occurring within the system of rivers containing one or more breaching dams. The time step is selected according to the following:

$$\Delta t = T_r/M \tag{14}$$

where T_r is the minimum time of rise of any hydrograph that has been specified at upstream boundaries or in the process of being generated at a breaching dam. M is user specified according to the following guidance:

$$M = 2.67 \left[1 + \mu n^{0.9}/(q^{0.1} S_o^{0.45})\right] \tag{15}$$

in which $\mu = 3.80$ (3.13 SI units), n is the Manning coefficient, q is the peak flow per unit channel width, and S_o is the channel bottom slope. M usually varies within the range, $6 \leq M \leq 40$, with M often assumed to be approximately 20.

The Δx computational distance step can be specified or automatically determined within the FLDWAV model according to the smaller of the two following criteria:

$$\Delta x \leq c T_r/20 \tag{16}$$

in which c is the bulk wave speed of the flood wave (hydrograph), and

$$\Delta x \leq L/N \tag{17}$$

where:

$$N = 1 + 2 \; |A_i - A_{i+1}|/\hat{A} \tag{18}$$

in which L is the distance between two adjacent cross sections differing from one another by approximately 50 percent or greater, A is the active cross sectional area, i and i+1 are index counters, $\hat{A} = A_{i+1}$ if $A_i > A_{i+1}$ (contracting reach) or $\hat{A} = A_i$ if $A_i < A_{i+1}$ (expanding reach), and N is rounded to the nearest integer value.

Robust Numerical Features: Additional numerical stability enhancements to the FLDWAV model include the following: (1) an automatic feature to prevent negative water surface elevations from being generated during the iterative solution of the implicit finite-difference Saint-Venant equations, (2) another feature prevents numerical overflow (the computed denominator of a fraction approaching zero), and (3) an optional low flow filter to prevent computed elevations and discharges to be retained if they are less than the initial steady flow conditions downstream of dams or less than the initial critical depths within reservoirs. The low flow filter should not be selected if the computed flows are expected to be less than the initial flows, e.g., locations where negative (reverse) flows occur or cases where the specified inflow hydrograph recedes below the initial flow at $t = 0$.

FLDWAV INPUT/OUTPUT

The input and output are in either English or metric (SI) units as specified at the beginning of a model run. The input conforms to a specified format with the last field of each line of data reserved for user defined identification information. The input is of the "batch" type. A special stand-alone menu-driven interactive input program is also scheduled to be available as a utility program to help the user develop the "batch" input file. Editing of existing data files is by either the menu-driven interactive program or a typical editor utility program. The latter is usually faster and more convenient for experienced FLDWAV users.

The FLDWAV output consists of numerical tables quite similar to those of the DAMBRK model and color-graphical displays. The graphical display features of FLDWAV are considerably advanced over the non-color line printer type provided in DAMBRK. FLDWAV's color-graphical display is a separate stand-alone program written in Microsoft C. It will display the following information as determined by the user through menu-driven interactive commands: (1) peak discharge profile; (2) peak water surface elevation profile; (3) multiple discharge hydrographs at user specified cross sections (original or interpolated); (4) multiple stage hydrographs at user specified cross sections; (5) computed stage-discharge relations (including loop effect) at user specified cross sections; (6) inflow discharge hydrograph; (7) downstream boundary hydrograph or stage-discharge rating curve; (8) multiple discharge and/or water surface profiles at selected times; (9) any cross section (original or interpolated) showing (a) the active, inactive, and floodplain portions of the section, (b) the maximum water surface elevation, and (c) the user specified flooding elevation or any other elevation of interest. A single or maximum of 15 profile(s) or hydrograph(s) may be displayed simultaneously.

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